VOLUME 80 SEPARATE No. 429

PROCEEDINGS

AMERICAN SOCIETY OF CIVIL ENGINEERS

APRIL, 1954

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

DISCUSSION OF PROCEEDINGS - SEPARATES

202, 286, 287, 300

ENGINEERING MECHANICS DIVISION

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

CONTENTS

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N.Y.

THE APPLICATION OF HEAVISIDE'S STEP-FUNCTION TO BEAM PROBLEMS

JOHN E. GOLDBERG, M. ASCE—Closure—The author wishes to thank the discussers for their critical and interesting comments and for their comparisons of the step-function method with other and more general methods, including those founded upon the operational calculus. Admittedly, these techniques, such as the Laplace Transform, make possible the formal solution of a much wider class of problems than the group treated by the author. Nevertheless, the essential simplicity of the step-function method, since it does not require the writing of a transform or an inverse, certainly has much in its favor for the type of beam problems treated in the paper.

Complications arise in certain other types of problems, and for this reason the elementary step-function method was purposely applied only to a simple class of beam problems. The introduction of the sine transform of the step-function, as suggested by Professor van Langendonck, permits the relatively straightforward extension of the step-function technique to a wide class of beam-column problems and, consequently, should prove extremely useful in refined analysis. The use of the Laplace Transform, suggested by Mr. Saunders, instead of the elementary step-function alone, likewise is a much more general method, readily extended to buckling and beam column problems. It is, of course, true that, as stated by Mr. Saunders, rather comprehensive tables are available for finding the Laplace Transform of a great number of functions and the necessary inverse transforms. The author feels, however, that so long as the problem is of the type treated in the paper, the elementary step-function is sufficient to provide a solution in an extremely direct manner, and there is no need to employ the more general methods. Lest this statement be interpreted as unprogressive, the author hastens to state that he, too, probably would use a method related to the Laplace Transform for the more general problem.

Professor Poggi has suggested and illustrated the use of the unit-impulse function in conjunction with the Laplace Transform, and has remarked upon the similarity of his method to that described by the author. In a sense, the unit-impulse function is a more flexible tool than the elementary step-function and, consequently, might prove even more attractive to some engineers. Professor Poggi's example, in which the Snitco-Hetenyi expansion is derived, is indeed interesting. One may remark, somewhat defensively, that expansion of the loading function, including concentrated loads, into a Fourier sine series is also extremely useful in many cases but, possibly, because of its transcendental nature, not quite as convenient as the Maclaurin series used by Professor Poggi.

The author concurs with Professor van Langendonck regarding the restriction upon the primitive, $F(x)$, appearing in Equation (3). It does indeed seem necessary that $F(O) = O$ for complete generality. This requirement is identically satisfied in the problems which appear in the paper.

DISCUSSION OF INELASTIC BEHAVIOUR OF REINFORCED CONCRETE MEMBERS SUBJECTED TO SHORT TIME STATIC LOADS PROCEEDINGS—SEPARATE NO. 286

B. BRESLER,[1] J.M., ASCE—Mr. Lee's application of the inelastic flexure theory to reinforced concrete members is a valuable addition to the literature on this subject. An interesting feature of Mr. Lee's paper is the assumption of parabolic stress-strain relationship for concrete in flexure and the verification of this assumption by beam tests. In deriving the relation between concrete stress and observed strains, Eq. (3), the author assumes an idealized stress-strain relationship for steel shown in Fig. 4 of the paper. For steel stresses between proportional limit and yield point this assumption may introduce significant errors if the actual steel stress strain diagram varies appreciably from that assumed.

The importance of the actual shape of steel stress-strain diagram becomes apparent if values of concrete stress corresponding to steel yield strain e_s are computed from both Eqs. (3) and (4). Since data in Table 1 of author's paper are limited to steel strains just below yield, the following estimated values will be used to illustrate the discrepancy between Eqs. (3) and (4). Consider yield strain e_s equal to 1615×10^{-6}, and an estimated value of concrete strain e_c equal to 2310×10^{-6}, from which the value of $(\frac{de_s}{de_c})$ is computed to be 0.40. Using these values f_c computed from Eq. (3) is 3600 psi, and that computed from Eq. (4) is 2130 psi. The discrepancy between these values of f_c indicates that at strains approaching yield author's Eqs. (3) and (4) are not valid.

Author's Eq. (2) can be rewritten in terms of steel stress f_s as follows:

$$\int_0^{e_c} f\,de = \frac{A_s E_s e_s e_c}{kbd} = \frac{p f_s e_c}{k} \qquad (2')$$

Substituting k in terms of e_s and e_c and differentiating both sides with respect to e_c, f_c is:

$$f_c = p\left[f_s\left(1 + \frac{de_s}{de_c}\right) + \frac{df_s}{de_c}(e_s + e_c)\right] \qquad (3')$$

Equation (3') is the general expression for f_c, and author's Eqs. (3) and (4) can be derived from it for the assumed shape of steel stress-strain diagram.

The flexural stress-strain curve for concrete can be obtained from beam test data using either Eq. (2') or Eq. (3'). Differentiation of Eq. (2') with respect to concrete strain determines the value of extreme fiber stress f_c corresponding to the extreme fiber strain e_c as follows:

1. Associate Prof. of Civ. Eng. Univ. of California, Berkeley, Calif.

$$\frac{d}{de}\int_{0}^{e_c} f\,de = f_c = \frac{d}{de}\left(\frac{p f_s e_c}{k}\right) \qquad (2'')$$

The stress-strain relation for a given range of strains can be determined from Eq. (2") using either graphical or numerical differentiation.

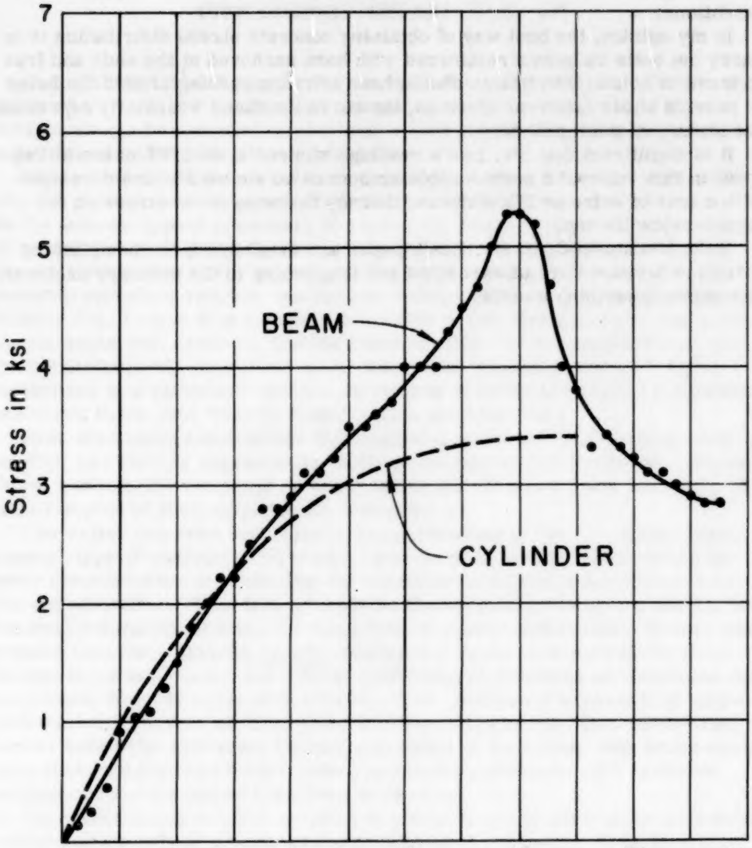
An experimental investigation of flexural stress-strain relationship based on Eq. (2") was carried out as part of M.S. Thesis[2] at the University of California, Berkeley. Three reinforced concrete beams 11 ft. 6 in. long having different concrete strengths, cross-sectional dimensions, and amounts of reinforcement were subjected to third-point loading and tested to failure. Strains were measured at three levels in the concrete and also in the steel reinforcement.

Stress-strain relationships for the three beams tested were determined from Eq. (2") using numerical differentiation procedure. Three 6 x 12 in. cylinders were cast with each beam and these were tested to determine the stress-strain relation in compression. In Fig. 15 flexural stress-strain relationship obtained from one of the beams is compared with the average of three compressive stress-strain curves obtained from cylinder tests. It can be seen that the flexural stress-strain diagram varies considerably from the cylinder stress-strain curve in the region of high stress.

It should be pointed out that calculation of flexural stress-strain relationship involves numerical differentiation which may be sensitive to the accuracy of experimental measurements. The experimental accuracy obtained in the tests described here is sufficient to allow the type of analysis proposed and therefore the curve shown in Fig. 15 is a reliable representation of test data obtained. Although test results described in this discussion seem to indicate that flexural stress-strain relationship for concrete may differ considerably from that obtained in compression, results of this limited investigation cannot be considered conclusive. Furthermore, the writer considers that the precise shape of the flexural stress-strain is primarily of academic interest as effects of plastic flow, shrinkage, size, previous history of stress and strain, state of stress, and stress gradient are neglected.

Mr. Lee's assumption of a parabolic stress-strain diagram allows prediction of theoretical values of strength and deformation of reinforced concrete members under short duration static loads in a relatively simple manner. However, additional experimental evidence is needed to establish the validity of the parabolic stress-strain relationship, particularly for predicting deformations of beams at loads approaching ultimate strength.

2. "Flexural Behavior of Reinforced Concrete Beams"—by Stuart H. Bartholomew, M.S. Thesis, University of California, Berkeley, California, October 1952.



STRESS-STRAIN CURVES FOR CONCRETE

A. J. ASHDOWN[3]— The results of Mr. Lee's estimation of concrete stresses in a beam would certainly be erroneous if obtained on an ordinary reinforced beam, since the interaction of the steel with the concrete is not as ordinarily assumed. The curve of stress obtained by J. M. Prentis was on a non-grouted prestressed beam and was probably nearer to the actual conditions.

In my opinion, the best way of obtaining concrete stress distribution is to carry out tests on beams reinforced with bars anchored at the ends and free to move in holes. The beams should have stirrups passing around the holes to prevent shear failures. Even so, the curve produced would only represent the history of the top fibre.

It is significant that Mr. Lee's readings stopped at 84 1/2% of the failing load; in this interval a considerable amount of strain would have developed with a loss of extreme fibre stress, thereby throwing more stress on the layers below the top.

If the fundamentals of Mr. Lee's paper are invalidated, the complicated formulae produced are unwarranted and frightening to the ordinary engineer, who wants something simple.

3. Civ. Eng. Dept., Imperial College, London, England.

DISCUSSION OF IMPULSIVE MOTION OF ELASTO-PLASTIC BEAMS PROCEEDINGS—SEPARATE NO. 287

P. S. SYMONDS[1]—The authors have made the valuable contribution in this paper of showing how the familiar normal mode type of analysis of elastic structures can be extended to problems where plastic flow must be considered in the transient response to a dynamic load.

There is no doubt that problems of dynamic loading of beams and frames into the plastic range are very difficult ones, so that the assumptions made by the authors appear necessary to render the problems tractable to analysis of moderate length and complexity. Their basic assumptions concern the physical properties of the beam, and can be summed up by saying that the moment-curvature relation has the same shape as the stress-strain diagram of their Fig. 1; thus it is assumed to consist of two straight lines, one at the elastic angle with slope EI, and the other parallel to the curvature axis and lying a distance M_o above this axis. When this "capacity moment" M_o is maintained at a particular section curvatures of arbitrarily large magnitudes can occur there, and "plastic hinge" action can take place.

Even with these assumptions the analytical and numerical work is often lengthy, and shorter approximate methods become highly desirable. The authors' methods for obtaining an upper bound (p. 13) and a lower bound (p. 15) are examples of such approximate methods.

The writer has been concerned with applications of the so-called "rigid-plastic" type of analysis of dynamical problems of structures, in which the basic simplification is made that deformations of elastic magnitude are neglected altogether. This type of analysis should give satisfactory results for the final deformations when the work done in plastic deformation is very much greater than the maximum elastic energy that could be stored in the beam. On this basis solutions for a variety of dynamical problems of structures have been found, in most cases with relative ease: problems of impact, of impulsive motion (initial velocities) and of various types of specified force-time curves have been solved for beams; a problem of a circular ring under concentrated side load has been treated; and certain problems of transverse bending of circular plates have been worked out.

The author's upper bound solution is obtained on the assumption that deformation occurs only at a hinge at the mid-point of the beam. This does not give the correct solution according to the rigid-plastic hypothesis. Their upper bound solution would be the correct rigid-plastic solution only for a beam strengthened in the interior of each half so that no deformation could occur except at the mid-point where the capacity moment is M_o. The correct rigid-plastic analysis for the uniform beam takes proper account of the plastic flow which occurs at sections other than the middle one. This analysis has been worked out recently by Cotter(1). The results for the central angle for the cases worked out by the authors are shown in Table I. It is seen that the final value of θ^* from the correct rigid-plastic analysis is considerably less than the final value of θ (from the authors' elasto-plastic analysis) and of θ_u (given by the authors' upper bound solution).

1. Associate Prof. of Eng., Brown University

Table I

Final Angle at Mid-Point of Free-Free Beam

$v_o = 11.1$					$v_o = 67.4$		
t	$\frac{\theta^*}{v_o} x10^3$	$\frac{\theta_u}{v_o} x10^3$	$\frac{\theta}{v_o} x10^3$	$\frac{\theta_\ell}{v_o} x10^3$	t	$\frac{\theta^*}{v_o} x10^2$	$\frac{\theta_u}{v_o}, \frac{\theta}{v_o}, \frac{\theta_\ell}{v_o} x10^2$
0	0	0	0	0	0	0	0
.05	0	.66	0	0	.25	0	.36
.10	.45	1.30	0	0	.75	.42	.93
.15	.93	1.80	0	0	1.0	.64	1.16
.20	1.31	2.18	0	0	1.25	.82	1.32
.25	1.57	2.44	.54	.40	1.50	.95	1.46
.30	1.72	2.57	1.38	.68	1.75	1.04	1.53
.35	1.77	2.63	1.97	.84	2.0	1.07	1.56
.40	1.77	2.63	2.20	.93	2.25	1.08	1.58
.45	↓	↓	2.20	.95	2.50	1.08	1.58
			↓	.95		↓	↓
				↓			

θ^* - Correct Plastic-Rigid Solution (Ref.[1])

θ_u - Upper Bound Solution

θ - Elastic-Plastic Solution

θ_ℓ - Lower Bound Solution

Bleich and Salvadori

Similarly, in their elasto-plastic solution for the uniform beam problem which the authors have worked out, they have taken proper account of plastic flow occurring at the mid-point of the beam, but have not considered possible violations of the yield condition at other points, at least in the paper. Again, the final deformations they compute would be correct only (in general) for a beam strengthened in the interior of each half-beam so that plastic flow could occur only at the mid-section, and hence their values of the central angle are in general too high. Although the authors state (page 9) that the solution for the elasto-plastic motion in terms of normal mode vibrations could be carried through for cases other than that where a single stationary hinge occurs, it would appear that the practical numerical difficulties would be very great. This perhaps explains why they have not included in the paper a discussion of methods for examining all cross sections of the beam to ensure that the capacity moment is not exceeded.

The authors have compared their single-hinge elasto-plastic solution with their single-hinge "rigid-plastic" solution, and find that for the value $v_o = 11.1$ the final angle values differ by about 20 per cent, while for the high value $v_o = 67.4$ the final results are identical. Since both their solutions assume plastic flow localized at a single hinge, the conclusions stated in their final paragraph seem untenable. In the first place, they have not compared a true rigid-plastic analysis with their elasto-plastic solution. More importantly, since their single-hinge "rigid-plastic" solution is certainly wrong, the observed agreement between the final angle computed by their upper bound method and that given by their elasto-plastic analysis shows that the elasto-

plastic solution likewise is erroneous. The agreement between their two solutions for large initial velocities is to be expected, since both apply to fictitious beams strengthened in such a way that plastic flow occurs only at the mid-section. The point is that this comparison gives no indication of the correctness of either solution, and while their elasto-plastic solution gives correct results at low velocities, both this and their upper bound solution are erroneous at large velocities.

The true rigid-plastic solution obtained by Cotter[1] can be expected to be correct at sufficiently high initial velocities. According to the criterion proposed by Lee and the writer[2] results obtained by the rigid-plastic type of analysis should be good approximations if the work done in plastic deformation greatly exceeds the maximum possible elastic strain energy that could be stored in the beam. The ratio of these two energies is about 1 in the authors' case where $v_0 = 11.1$, and about 36 in the case of $v_0 = 67.4$. Since valid elasto-plastic solutions for a uniform beam problem have not yet been obtained one cannot yet say how big this ratio must be in order for the rigid-plastic analysis to give satisfactory results. Comparisons have been made for simple mass-spring systems, however[3], [4]. That given in[4] indicates that when the energy ratio is higher than 20 the rigid-plastic type of analysis gives results accurate to within 5 per cent in an initial velocity problem. It seems fairly likely from this result that the rigid-plastic solution obtained by Cotter for the initial velocity 67.4 is reasonably accurate.

Reference[4] deals with perhaps the simplest impulsive motion problem of a uniform beam, namely that of a simply supported span with an initial velocity proportional to $\sin \frac{\pi}{\ell} x$, ℓ being the span length. It is easy to verify directly in this problem that a "single-hinge" type of elasto-plastic analysis is erroneous even for fairly small initial velocities. Even in this problem, however, it seems to be very difficult to obtain the correct elasto-plastic solution for arbitrary initial velocities. It would be most valuable to have the benefit of the authors' experience in dealing with such problems.

REFERENCES

1. "Plastic-Rigid Analysis of a Beam under a Symmetric Impulsive Loading" by B. A. Cotter. Technical Report No. All-104 of Brown University to Office of Naval Research under Contract N7onr-35801, in preparation.

2. "Large Plastic Deformations of Beams under Transverse Impact" by E. H. Lee and P. S. Symonds, Jour. Appl. Mech. Vol. 19, No. 3, 1952, pp. 308-315.

3. "The Response of Simple Structures to Dynamic Loads" by Nancy Brazell Brooks and N. M. Newmark, Technical Report of University of Illinois to Office of Naval Research under Contract N6ori-071(06); Structural Research Series No. 51, April 1953.

4. "On the Elastic-Plastic and Rigid-Plastic Deformations of a Beam under Impulsive Loading" by J. A. Seiler, B. A. Cotter, and P. S. Symonds, Technical Report No. UERD-1 of Brown University to Norfolk Naval Shipyard under Contract N189s-92558, in preparation.

DISCUSSION OF DESIGNING ALUMINUM ALLOY MEMBERS FOR COMBINED END LOAD AND BENDING PROCEEDINGS—SEPARATE NO. 300

R. R. McCALLEY, Jr.[1] J. M. ASCE.—Unsymmetrical cases occur sufficiently often in practice that a generalized form of Eq. (6) may be of interest. Eq. (6) applies to the case in which the constant bending moment lies in one of the principal planes of the cross section. Consider the more general case in which the x and y axes do not coincide with the principal axes. For a beam subject to a constant bending moment about the x axis, the critical moment, M_{cr}, can be expressed as:[2]

$$M_{cr} = \frac{\pi^2 E I_1 I_2}{I_x (KL)^2} \left[e \pm \sqrt{e^2 + \frac{G J I_x (KL)^2}{\pi^2 E I_1 I_2} \left(1 + \frac{\pi^2}{(KL)^2} \frac{E C_s}{G J} \right)} \right] \qquad \text{(a)}$$

where I_1 is the major moment of inertia, I_2 is the minor moment of inertia, and I_x is the moment of inertia about the x axis.

However, the generalized definition of e is now given by:

$$e = y_t - y_s \qquad \text{(b)}$$

where y_t is the y coordinate of a geometric point designated the "twist center," and y_s is the y coordinate of the shear center. The "twist center" is the point about which a beam in pure torsion rotates. In symmetrical cases both the "twist center" and the shear center coincide with the centroid of the cross section. The "twist center" coordinates (x_t, x_t) may be computed from the formulas:[2]

$$x_t = \frac{1}{2\, I_1\, I_2} \left(I_x \int \rho_a^2\, x\, dA - I_{xy} \int \rho_a^2\, y\, dA \right) + x_a \qquad \text{(c)}$$

$$y_t = \frac{1}{2\, I_1\, I_2} \left(I_y \int \rho_a^2\, y\, dA - I_{xy} \int \rho_a^2\, x\, dA \right) + y_a \qquad \text{(d)}$$

where ρ_a is the radius vector from some convenient reference point (x_a, y_a) to a point (x, y) on the cross section and the integrations are taken over the whole area, A, of the cross section.

The critical stress f_B may then be determined from the formula:

$$f_B = M_{cr} \left(I_y\, y - I_{xy}\, x \right) / \left(I_1\, I_2 \right) \qquad \text{(e)}$$

1. Engineer, Applied Physics Laboratory, Johns Hopkins University, Silver Spring, Maryland.

2. "Contributions to the Theory of Combined Flexure and Torsion" by R. B. McCalley, Jr., Thesis, Cornell University, June 1952.

The x and y coordinates of the extreme fiber must be chosen by inspection for the worst case. In many instances, it may be convenient to eliminate the product term $I_1 I_2$ in the foregoing equations by means of the identity:

$$I_1 I_2 = I_x I_y - I_{xy}^2 \qquad \text{(f)}$$

In the event that the x axis coincides with the major axis of the cross section, $I_x = I_1$, $I_y = I_2$, $I_{xy} = 0$, and Eq. (a) reduces to Eq. (6). Goodier[3] has previously called attention to the fact that the $\pm$ sign before the radical term indicates that the beam has one critical moment when bent downward but another when bent upward. The quantity e is equal to one-half of Timoshenko's β_1.[4] Notice that the expression given by Timoshenko is referred to the principal axes of the section.

---

3. Reference 9, page 10.

4. "Theory of Bending, Torsion, and Buckling of Thin-Walled Members of Open Cross Section" by S. P. Timoshenko, Journal of the Franklin Institute, May 1945, page 356.

PROCEEDINGS-SEPARATES

The technical papers published in the past twelve months are presented below. Technical-division sponsorship is indicated by an abbreviation at the end of each Separate Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways (WW) divisions. For titles and order coupons, refer to the appropriate issue of "Civil Engineering" or write for a cumulative price list.

VOLUME 79 (1953)

MAY: 189(HY), 190(HY), 191(CP) & (AT), 192(SM), 193(HY), D-129(PO), D-138(CP), D-145(ST).

JUNE: 194(CP) & (AT), 195(SM), 196(CP) & (AT), 197(HY), 198(ST), 199(EM), D-134(HY), D-141(HY).

JULY: [a] 200(SM)[b], 201(ST)[b], 202(EM)[b], 203(SM)[b], 204(AT)[b], 205(EM)[b], 206(ST)[b], 207(SA)[b], 208(SA)[b], 209(ST)[b], 210(SU)[b], 211(EM)[b], 212(SU)[b], 213(IR)[b], 214(HW)[b], 215(SM)[b], 216(ST)[b], 217(ST)[b], 218(ST)[b], 219(ST)[b], 220(SM)[b], 221(HW)[b], 222(SM)[b], 223(EM)[b], 224(EM)[b], 225(EM)[b], 226(CO)[b], 227(SM)[b], 228(SM)[b], 229(IR)[b].

AUGUST: 230(HY), 231(SA), 232(SA), 233(AT), 234(HW), 235(HW), 237(AT), 238(WW), 239(SA), 240(IR), 241(AT), 242(IR), 243(ST), 244(ST), 245(ST), 246(ST), 247(SA), 248(SA), 249(ST), 250(EM)[c], 251(ST), 252(SA), 253(AT), 254(HY), 255(AT), 256(ST), 257(SA), 258(EM), 259(WW).

SEPTEMBER: 260(AT), 261(EM), 262(SM), 263(ST), 264(WW), 265(ST), 266(ST), 267(SA), 268(CO), 269(CO), 270(CO), 271(SU), 272(SA), 273(PO), 274(HY), 275(WW), 276(HW), 277(SU), 278(SU), 279(SA), 280(IR), 281(EM), 282(SU), 283(SA), 284(SU), 285(CP), 286(EM), 287(EM), 288(SA), 289(CO).

OCTOBER: [d] 290(all Divs), 291(ST)[c], 292(EM)[c], 293(ST)[c], 294(PO)[c], 295(HY)[c], 296(EM)[c], 297(HY)[c], 298(ST)[c], 299(EM)[c], 300(EM)[c], 301(SA)[c], 302(SA)[c], 303(SA)[c], 304(CO)[c], 305(SU)[c], 306(ST)[c], 307(SA)[c], 308(PO)[c], 309(SA)[c], 310(SA)[c], 311(SM)[c], 312(SA)[c], 313(ST)[c], 314(SA)[c], 315(SM)[c], 316(AT), 317(AT), 318(WW), 319(IR), 320(HW).

NOVEMBER: 321(ST), 322(ST), 323(SM), 324(SM), 325(SM), 326(SM), 327(SM), 328(SM), 329(HW), 330(EM)[c], 331(EM)[c], 332(EM)[c], 333(EM)[e], 334(EM), 335(SA), 336(SA), 337(SA), 338(SA), 339(SA), 340(SA), 341(SA), 342(CO), 343(ST), 344(ST), 345(ST), 346(IR), 347(IR), 348(CO), 349(SM), 350(HW), 351(HW), 352(SA), 353(SU), 354(HY), 355(PO), 356(CO), 357(HW), 358(HY).

DECEMBER: 359(AT), 360(SM), 361(HY), 362(HY), 363(SM), 364(HY), 365(HY), 366(HY), 367(SU)[e], 368(WW)[e], 369(IR), 370(AT)[e], 371(SM)[e], 372(CO)[e], 373(ST)[e], 374(EM)[e], 375(EM), 376(EM), 377(SA)[e], 378(PO)[e].

VOLUME 80 (1954)

JANUARY: 379(SM)[e], 380(HY), 381(HY), 382(HY), 383(HY), 384(HY)[e], 385(SM), 386(SM), 387(EM), 388(SA), 389(SU)[e], 390(HY), 391(IR)[e], 392(SA), 393(SU), 394(AT), 395(SA)[e], 396(EM)[e], 397(ST)[e].

FEBRUARY: 398(IR)[f], 399(SA)[f], 400(CO)[f], 401(SM)[f], 402(AT)[f], 403(AT)[f], 404(IR)[f], 405(PO)[f], 406(AT)[f], 407(SU)[f], 408(SU)[f], 409(WW)[f], 410(AT)[f], 411(SA)[f], 412(PO)[f], 413(HY)[f].

MARCH: 414(WW)[f], 415(SU)[f], 416(SM)[f], 417(SM)[f], 418(AT)[f], 419(SA)[f], 420(SA)[f], 421(AT)[f], 422(SA)[f], 423(CP)[f], 424(AT)[f], 425(SM)[f], 426(IR)[f], 427(WW)[f].

APRIL: 428(HY)[e], 429(EM)[e], 430(ST), 431(HY), 432(HY), 433(HY), 434(ST).

---

a. Beginning with "Proceedings-Separate No. 200," published in July, 1953, the papers were printed by the photo-offset method.
b. Presented at the Miami Beach (Fla.) Convention of the Society in June, 1953.
c. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
d. Beginning with "Proceedings-Separate No. 290," published in October, 1953, an automatic distribution of papers was inaugurated, as outlined in "Civil Engineering," June, 1953, page 66.
e. Discussion of several papers, grouped by divisions.
f. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1954

PROCEEDINGS OF THE SOCIETY